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Proposed Limit State Strength Evaluation of Existing Reinforced-Concrete Bridges

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ABSTRACT

Because of several catastrophic bridge failures, bridge safety has been emphasized during the past decade. As a result there has been a concerted effort to develop and disseminate procedures for systematic bridge inspection and rating. Although bridges with concrete superstructures rarely fail catastrophically, gradual deterioration and increased loads can affect their structural capacity. Existing procedures for inspecting and rating bridges with concrete superstructures are limited. A summary of a methodology proposed for rating reinforced-concrete bridges is presented. The methodology was developed in the first phase of an NCHRP project to improve strength evaluations of existing reinforced-concrete bridges. The methodology is presented in a limit-states format by using approximate load and resistance factors. By using this format a basis is provided on which probability theory and engineering judgment can be rationally combined to allow for independent consideration of each of the major variables that can affect the determination of the load capacity of a bridge. This methodology includes consideration of the level of effort in maintenance and inspection, the degree of load-limit enforcement, the quality of construction, the refinement used in simulating the bridge, the effects of deterioration on the load-carrying capacity, and the degree of refinement in determining the load-distribution factors.

Currently, the procedure for evaluating reinforced-concrete bridges in the United States is based on AASHTO guidelines published in the Manual for Maintenance Inspection of Bridges (1). Experience has demonstrated that the structural capacity of reinforced-concrete bridges usually exceeds the capacity calculated by the conventional techniques presented in this manual.

Many engineers recognize the built-in conservatism in the current approach to the evaluation of bridge strength. Factors that tend to cause the capacity of reinforced-concrete bridges to be underestimated include

1. Material strengths that exceed nominal values used for evaluation,
2. Conservative assumptions used in calculating structural resistance (i.e., zero tension in concrete),
3. Interaction of structural components in resisting and distributing the loads,
4. Structural redundancies, and
5. Overestimation of the loads.

INTRODUCTION

To make improvements in the bridge evaluation process that will lead to more realistic evaluations while still preserving public safety requires a rational consideration of each of the five factors previously mentioned. One method for making such improvements is through a limit-states approach based on probabilistic concepts. This approach was used in the recently developed Clause 12 of the Canadian Standards Association Bridge Code (2-4).

The proposed methodology for evaluating existing bridges incorporates such a limit-states approach. Although the methodology represents a significant change in the current philosophy, from the user's point of view it is similar to the current AASHTO load-factor approach. It should be emphasized that the user need not understand the statistical basis of the methodology to effectively apply it to the evaluation of a structure. The specific values included in this paper for load and resistance factors are based on a combination of preliminary statistical data and engineering judgment. These values should be considered as preliminary and are intended only to illustrate the overall approach of the methodology.

DEFINITION OF PROPOSED LIMIT-STATES EVALUATION

When a structure or structural element becomes unfit for its intended purpose, it is said to have reached its limit state (5,6). Limit states fall into two categories: safety limit states and serviceability limit states. Structural reliability is the probability that a given structure will perform satisfactorily by not reaching its limit state over a specified time period.

Safety limit states (i.e., ultimate limit states) correspond to the ability of the structure or structural component to support the applied loads. Serviceability limit states either restrict the normal use of a bridge or affect its durability. The acceptable level of structure reliability will vary, depending on the type of limit state used in the calculations.

EVALUATION PROCESS

The limit-states evaluation process (see Figure 1) described in this paper consists of the following steps.

Step 1--collection of information: field inspection, office records, and special testing;
Step 2--selection of rating vehicle: standard vehicle, overload vehicle, and special permit vehicle;
Step 3--analysis: identification of critical failure mode(s), determination of nominal load effects, and determination of nominal resistance;
Step 4--selection of load and resistance factors: charts and engineering judgment; and
Step 5--determination of rating factors.

The results of the structural strength evaluation may be used to determine restrictions on the use of the bridge by normal traffic (load limit posting), the maximum weight of the occasional overload vehicle allowed to mix with normal traffic (unsupervised overload permit), or the absolute maximum weight of any vehicle allowed on the bridge under controlled circumstances (supervised overload permit). In addition, a substandard live-load capacity may also be justification for future repairs or replacement or both.

PROPOSED RATING EQUATION

The basic structural engineering equation states that the resistance of a structure must equal or exceed the demand placed on it by loads. Stated mathematically,

$$R \geq \sum_{k=1}^{n} Q_k \qquad (1)$$

where R is the resistance and Q_k is the effect of load k. The solution of this simple equation encompasses the whole art and science of structural engineering, including the disciplines of strength of materials, structural analysis, and load determination. This equation applies to design as well as to evaluation. In structural evaluations the objective is to select the maximum allowable live load. In the case of bridge evaluations, this usually means the maximum vehicle weight.

Any rational and tractable approach to the analytical solution of the basic structural engineering equation requires that the modes of failure be identified to establish the resistance. The location, types, and extent of the critical failure modes must be determined. The equation must be solved for each of these potential failure modes.

Because neither the resistance nor the load effect can be established with certainty, safety factors must be introduced that give adequate assurance that the limit states are not exceeded. This may be done by stating the equation in a load and resistance factor format.

Separate load or resistance factors that will account for each of the major sources of uncertainty may be introduced to the equation. The basic rating equation used in the proposed approach is simply a special form of the basic structural engineering equation, with load and resistance factors introduced to account for uncertainties that apply to the bridge evaluation problem; that is,

$$RF = \left[(\phi R/\alpha) - \sum_{i=1}^{m} \gamma_i^D D_i - \sum_{j=1}^{n} \gamma_j^L L_j (1.0 + I)\right] / [\gamma_R^L L_R (1.0 + I)] \qquad (2)$$

where

- RF = rating factor (the portion of the rating vehicle allowed on the bridge),
- ϕ = capacity reduction factor,
- m = number of elements included in the dead load,
- α = simulation factor,
- R = nominal resistance,
- n = number of live loads other than the rating vehicle,
- γ_i^D = dead-load factor for element i,
- D_i = nominal dead-load effect of element i,
- γ_j^L = live-load factor for live load j other than the rating vehicle(s),
- L_j = nominal traffic live-load effects for load j other than the rating vehicle(s),
- γ_R^L = live-load factor for rating vehicle,
- L_R = nominal live-load effect for rating vehicle, and
- I = live-load impact factor.

Equation 2 should be evaluated for both the safety and the serviceability limit states. The following subsections discuss the philosophy and parameters considered for each of the variables in this proposed rating equation.

Simulation Factors

The capacity of an existing bridge is evaluated by simulating the hypothetical failure scenario that is most likely to occur within the life expectancy of the bridge. A mathematical model, field inspection, test results, and engineering judgment are typically used in this simulation. A simulation factor (α) is introduced to account for the refinement or level of effort used in simulating the actual failure scenario. Three levels of simulation were selected initially (see Table 1). Requirements for field inspection, analysis, and the rater/checker must be met or exceeded before the tabulated simulation factors can be used.

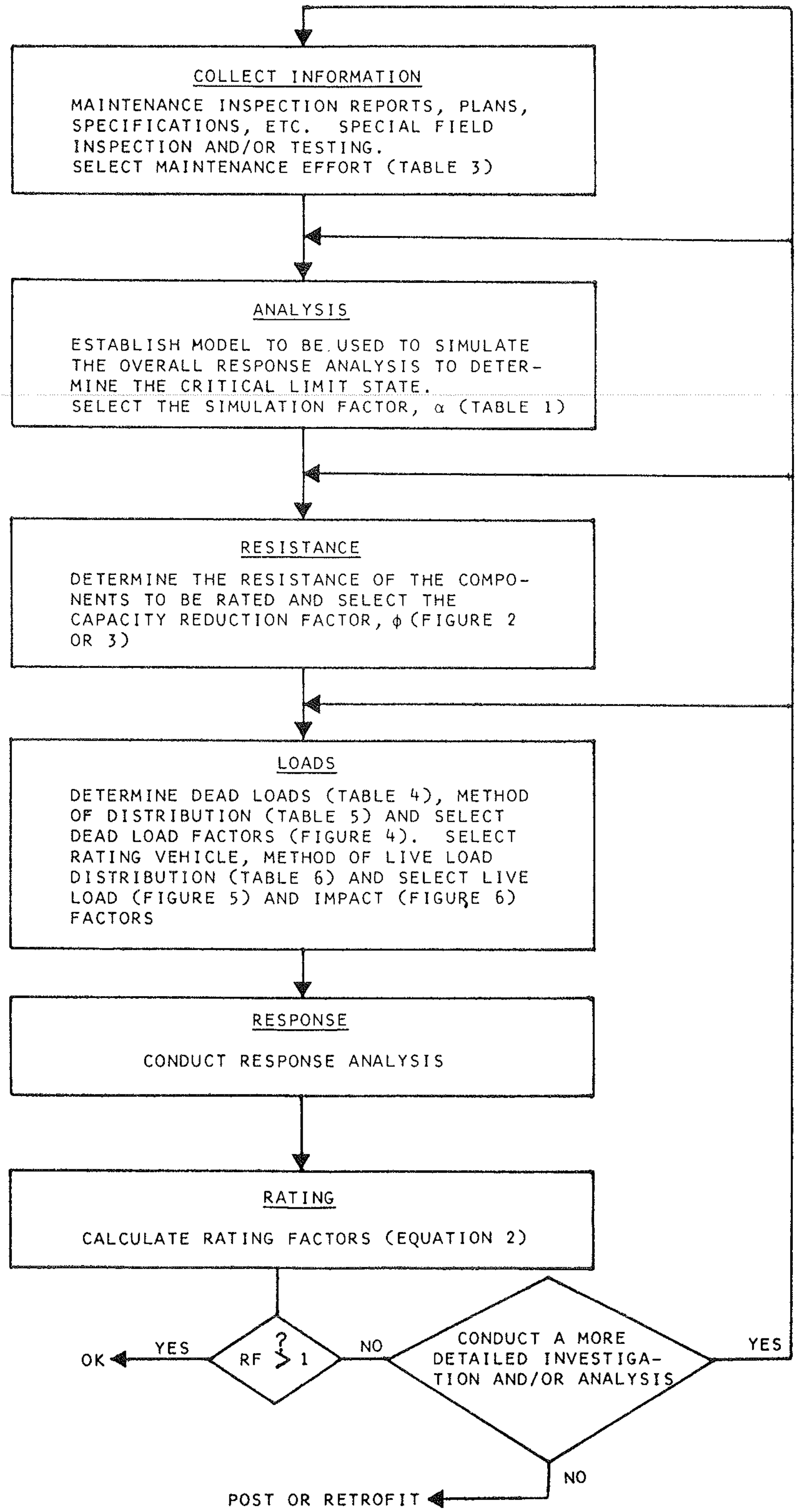

FIGURE 1 Flowchart of evaluation process.

TABLE 1 Simulation Factors

Factor	Field Inspection	Analysis	Rater/Checker
0.95	Detailed: conducted by rater or checker; will include determination of location and extent of deterioration; structural dimension verified by field measurement; some field testing may be required	Detailed: some secondary structural effects considered (finite element, grid, orthotropic plate, three-dimensional space frame in conjunction with influence surfaces)	Both rater and checker are professional engineers with at least 1 year of experience in rating bridges
1.0	Normal biennial maintenance inspection conducted by professional engineer or qualified inspector; structural dimensions taken from plans; rater to review maintenance records	Detailed: only primary effects considered (two-dimensional plane frame in conjunction with AASHTO live-load distribution factors)	Either rater or checker is professional engineer with at least 1 year of experience in rating bridges
1.05	Normal biennial maintenance inspection; some structural dimensions assumed; maintenance records not reviewed	Approximate: simplified idealization of structure that includes only the most critical primary effects (neglect structure continuity, and so forth)	Rater is a professional engineer with at least 1 year of experience; no independent check conducted

Resistance

The determination of the structural resistance (R) of a structure or structural component is one of the primary tasks in the evaluation process. In a limit-states approach it is necessary to define the limit states at which resistance will be determined. Regardless of the material or structure type, these limit states should provide for similar structural performance.

Safety Limit States

Safety limit states are those states that correspond to the maximum load-carrying capacity of a structure or component. These limit states should be set at a low probability of occurrence because failure of the structure or component can lead to loss of life as well as to major financial losses. Safety limit states include

1. Loss of equilibrium of all or part of the structure considered as a rigid body (e.g., overturning, sliding, uplift);
2. Loss of load-bearing capacity of members because of insufficient material strength, buckling, fatigue, fire, corrosion, or deterioration;
3. Overall instability of the structure (e.g., P delta effect, wind flutter, seismic motions); and
4. Extremely large deformation (e.g., transformation into a mechanism).

In the case of reinforced-concrete structures subjected to traffic live loads, the safety limit state is assumed to occur when an individual component such as a girder reaches its ultimate capacity and forms a plastic hinge. In most cases this state does not present a serious threat to safety. The actual threat to safety occurs when enough plastic hinges are formed within the structure to result in a collapse mechanism. Many studies have indicated that this will normally occur at a loading significantly greater than the load at which the first plastic hinge was formed. This is because most reinforced-concrete structures have a high level of structural redundancy. Therefore, what is currently defined as the safety limit state would in most cases be more appropriately called a severe damage limit state.

The nominal resistance of reinforced-concrete members at the safety limit state is the ultimate strength of any given member. Strength calculations should take into consideration the observable effects of deterioration, which may include (but are not limited to) loss of concrete or steel cross-sectional area, loss of composite action, or reduced material strengths.

It is proposed that the strength of sound concrete shall be assumed to be equal either to the values taken from the plans and specifications or to the average of the construction test values. When neither of these values are available, the ultimate stress of sound concrete may be assumed to be 3,000 psi. A reduced ultimate stress should be assumed for unsound or deteriorated concrete, unless evidence to the contrary is discovered by field testing.

To allow for undetected structural weaknesses, it is proposed that the area of tension steel to be used in computing the ultimate flexural strength of reinforced-concrete members should not exceed 75 percent of the reinforcing required for a balanced condition. The steel yield stresses proposed for various types of reinforcing steel are given in Table 2.

TABLE 2 Yield Stress of Reinforcing Steels

Reinforcing Steel	Yield Stress, F_y (psi)
Unknown steel (before 1954)	33,000
Structural grade	36,000
Intermediate grade and unknown after 1954 (grade 40)	40,000
Hard grade (grade 50)	50,000
Grade 60	60,000

Serviceability Limit States

Serviceability limit states either restrict the normal use of the bridge or affect its durability. These limit states include

1. Excessive deflection or rotation that affects the use or appearance of the structure or of nonstructural components;
2. Excessive local damage (e.g., cracking, splitting, spalling, local yielding, slip of connection) that affects the use, durability, or appearance of the structure; and
3. Excessive undesirable vibrations.

The most important serviceability limit states in a bridge evaluation are those that tend to affect the durability of the structure and shorten its useful life. Two types of serviceability failures are considered critical for reinforced concrete.

One of these critical serviceability failures is fatigue in the reinforcing steel. This will occur when a large number of repetitive live loads result in large variations in the steel stresses. The critical number of load repetitions is only likely to occur as a result of normal traffic. Because evaluation of the serviceability limit state for fatigue is not used to restrict live loadings, its primary function is to alert the engineer to a potential problem that will warrant more frequent field inspections.

TABLE 3 Maintenance Effort (safety limit states)

Inspection	Preventive Maintenance	Repair	Maintenance Effort
Annual inspections by professional engineer involved in performing or checking structural strength evaluation	Steps taken to prevent further damage	Within 5 years, when capacity is currently impaired or when it may possibly become impaired	1
Annual inspections by professional engineer or qualified inspector	None	None	2
No special inspection	None	None	3

Crack control is the other critical serviceability limit state considered in the evaluation of existing reinforced-concrete bridges. The effect that crack width has on the rate of deterioration of structures exposed to severe environments is still unknown. Nevertheless, there is some concern that excessive crack width can cause an increase in the rate of deterioration, although several other factors not associated with the level of live loading also play a role.

The allowable steel stress limitations are based on fatigue and crack control requirements as described in AASHTO Sections 1.5.38 (Fatigue Stress Limits) and 1.5.39 (Distribution of Flexural Reinforcement) (7). The following conditions are recommended for serviceability limit states:

1. Restrictions of normal traffic (i.e., below posted weight limits) should not be required to maintain serviceability;
2. Fatigue stress limitations should not be considered for occasional overload trucks; and
3. Frequent inspections should be conducted on bridges subject to live loadings that produce steel stresses beyond the recommended allowable stresses for serviceability.

Capacity Reduction Factor

A capacity reduction factor (ϕ) is included in the basic rating equation to account for variation in the calculated resistance. It takes into consideration the dimensional variations of the structure, differences in material properties, future deterioration, and potential inaccuracies in the theory for calculating resistance.

The capacity reduction factor also accounts for variations in inspection and maintenance efforts that limit the ability of the inspector to detect or prevent future deterioration or distress that can potentially result in losses in live-load capacity. The maintenance effort for bridges that show signs of deterioration distress is categorized into three proposed levels (Table 3). The inspection, preventive maintenance, and repair conditions must be met or exceeded before the tabulated value for the maintenance factor is used. Note that maintenance efforts on structures with no observable deterioration or distress, which are inspected biennially by a professional engineer or by a qualified inspector, shall be classified as maintenance effort 1. Maintenance efforts on bridges with observable deterioration shall be classified as either maintenance effort 2 or 3, depending on the amount of deterioration and the frequency of inspection.

The proposed capacity reduction factors for safety limit states shall be taken from Figure 2 for flexure and from Figure 3 for shear. The capacity reduction factor for serviceability limit states shall be equal to 1.0.

Dead-Load Factors

Dead loads, which shall be determined from dimensions on the plans or from field measurements, shall include the weights of each of the permanent parts and appendages of the bridge. Partial dead-load factors are proposed to reflect both the various degrees of control used in producing the structural and nonstructural components of the bridge and the degree of analytical refinement used to determine the distribution of dead load to the structural components. The minimum unit weights of materials to be used in computing the dead load are taken from Table 4.

The effort used to determine dead-load distribution is categorized into three proposed levels of refinement (Table 5). Once the level of refinement for the dead-load distribution has been selected, separate dead-load factors (γ^D) are obtained for each type of component in the bridge. The dead-load factors proposed for use in the evaluation of safety limit sates are shown in Figure 4. Dead-load factors for serviceability limit states shall be equal to 1.0.

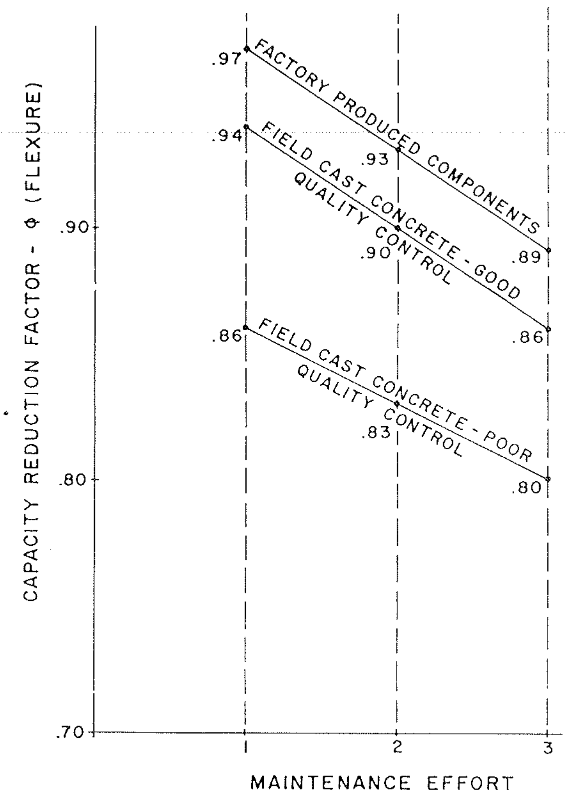

FIGURE 2 Capacity reduction factors—flexure.

Live-Load Factors

Highway vehicles come in a wide variety of sizes and configurations. No single vehicle can accurately reflect the effects of all of these vehicles. Because it is necessary to limit the number of vehicle configurations to a manageable level in order to

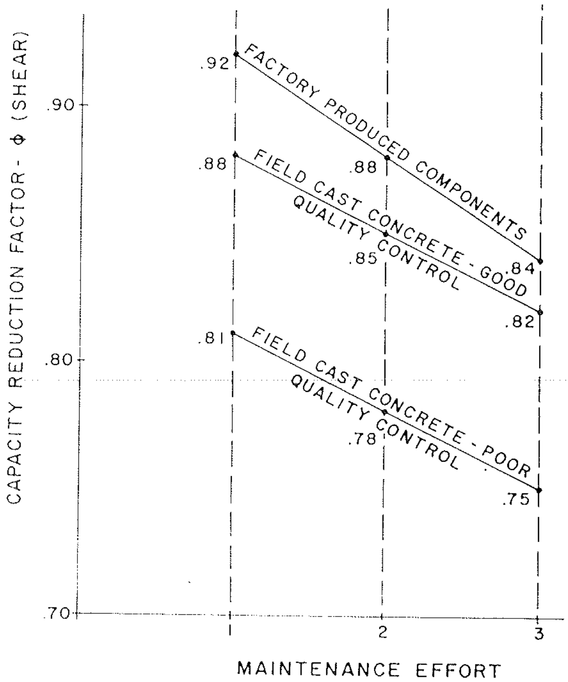

FIGURE 3 Capacity reduction factors—shear.

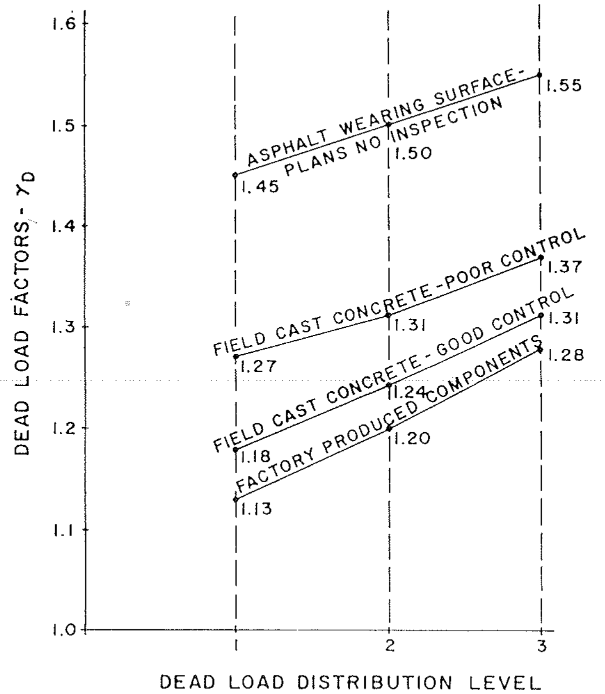

FIGURE 4 Dead-load factors.

keep the evaluation process from becoming too cumbersome, the effect of the actual traffic live loads will vary from predicted values. This variation will usually be greater than the variation in dead-load effect.

Live-load factors (γ^L) are used in the evaluation to account for variations in the maximum live-load effects that are likely to occur during the life of the bridge. Because the effect must be measured in relation to the maximum weight of the vehicles actually allowed on the bridge, it is affected by the amount of control exercised over weight limits. If load limits are strictly enforced or if there is close control of the types of vehicles granted overload permits, the variation in maximum live load will be less, and a smaller live-load factor is justified. If, on the other hand, load limits cannot be adequately enforced and violations are likely, then a higher live-load factor must be used to provide for higher overloads.

The degree of refinement or sophistication used to determine the distribution of live loads to the load-carrying components is also included in the live-load factor. The three proposed levels of refinement are given in Table 6.

The live-load factors proposed for use in the evaluation of safety limit states are shown in Figure 5. Live-load factors for serviceability limit states shall be equal to 1.0.

TABLE 4 Dead-Load Unit Weights

Material	Unit Weight (lb/ft^3)
Asphalt surfacing	144
Concrete, plain or reinforced	150
Steel	490
Cast iron	450
Timber (treated or untreated)	50
Earth (compacted), sand, gravel, or ballast	120

TABLE 5 Dead-Load Distribution Levels

Level	Description
1	Grillage analogy, orthotropic plate, finite element
2	Loadings from tributary areas in which reactions are computed by including the continuity of the structure
3	Loadings from tributary areas in which reactions are computed by assuming simple supports

TABLE 6 Live-Load Distribution Levels

Level	Description
1	Grillage analogy, orthotropic plate theory, finite element, or specially prepared influence surfaces developed by using one of these methods
2	Load distributions based on formulas that have been derived for specific loads, such as AASHTO design live-load distribution factors for AASHTO design loads
3	Load distributions based on formulas that are not specifically intended for the loading under consideration or load distributions based on simple support reactions

Impact Factors

It is proposed that the dynamic effects of moving live loads shall be included in the evaluation of both the safety and serviceability limit states. As part of the development effort for the Ontario bridge code (8), comprehensive studies were conducted on the dynamic effects of moving vehicles. The findings from these studies led to the development of impact factors (I) that are dependent on the dynamic frequency of the bridge deck. The method for calculating impact in the proposed rating equation is specified in the Ontario bridge code.

The impact factor for components of deck slabs with designs governed by a single-axle or dual-axle unit shall not be less than 0.40. In addition, the impact factor for the following items shall not be

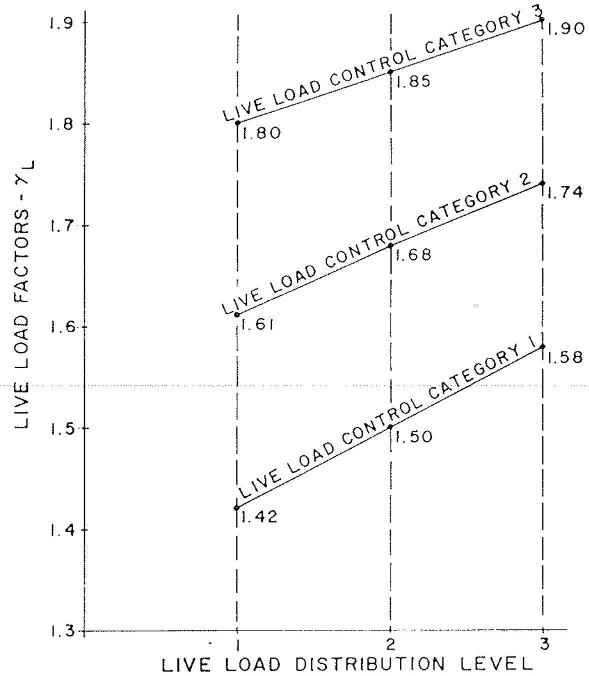

FIGURE 5 Live-load factors.

less than 0.35: (a) floor beams supporting deck slabs, (b) other beams with spans less than 40 ft, and (c) slabs with spans less than 40 ft.

The load factor for each of the main longitudinal components other than those previously mentioned is taken from Figure 6 as a function of the first flexural frequency of the given component. The first flexural frequency may be determined from a dynamic analysis, tests, or an approximate formula (see Equation 3). The impact factor shall be the maximum value obtained from Figure 6 for any frequency within ±10 percent of the calculated value.

For the purpose of determining the impact factor, the first flexural frequency shall be calculated by using the static properties of the materials. In the absence of advanced mathematical modeling techniques or tests, the following approximate formula may be used to obtain the frequency:

Frequency (in hertz) = 400/span (in feet) (3)

ILLUSTRATIVE EXAMPLE

This example is intended to illustrate the application of the proposed methodology. Note that, as mentioned previously, the factors included in the proposed procedure are based on limited statistical data and are only included to illustrate the overall procedure. The bridge selected for this example is a single-span, T-beam structure; its dimensions and member properties are shown in Figure 7. The bridge was constructed in 1925, and it is assumed in this illustrative example that a thorough field inspection revealed insignificant deterioration. Given this assumption, the bridge is to be evaluated for normal traffic loadings. Rating factors will only be calculated for flexure in the interior girders for the following vehicles: AASHTO HS20-44; and legal vehicles: type 3, type 3S2, and type 3-3 (1).

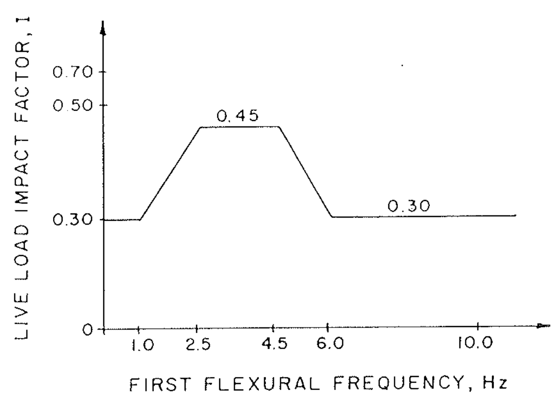

FIGURE 6 Diagram of impact factor.

Simulation Factor

The bridge has been inspected by a qualified inspector as part of the normal biennial inspection. The analysis was performed by using a two-dimensional idealization of the bridge in conjunction with AASHTO load-distribution factors. This evaluation was performed by registered engineers experienced in bridge evaluation. By using the data in Table 1, the appropriate simulation factor is 1.0.

Resistance: Safety Limit States

Concrete

The field inspection revealed that the concrete was sound. Because the plans contained information on the design concrete strength and because construction records are not available, assume that f'_c = 3,000 psi.

Reinforcing Steel

Because the structure was built in 1925 and the reinforcing steel type is unknown, assume f_y = 33,000 psi (Table 2). To calculate the ultimate moment capacity, the following properties were determined from the dimensions on the bridge plans:

1. Gross steel area: $A_s = 6.89$ in.^{2};
2. Depth of steel: d = 2.22 ft;
3. Depth of concrete compression block: a = 1.14 in. (0.095 ft); and
4. Ultimate moment capacity (resistance): $R = A_s f_y[d - (a/2)]$ = 494 kip-ft.

Resistance: Serviceability Limit States

Because f_y = 33,000 psi is less than the f_y = 40,000 psi limit, serviceability limit states will not be critical. Therefore, no calculations for fatigue or cracking are made.

Capacity Reduction Factor

No deterioration is present, the quality of construction appears satisfactory, and the bridge is on the biennial inspection program. Therefore, the capacity reduction factor (ϕ) taken from Figure 2 is 0.94.

Dead-Load Factors

A summary of the calculated dead loads is as follows:

Concrete section = 0.87 kip/ft,
Rail = 0.22 kip/ft, and
Asphalt concrete (AC) overlay = 0.40 kip/ft.

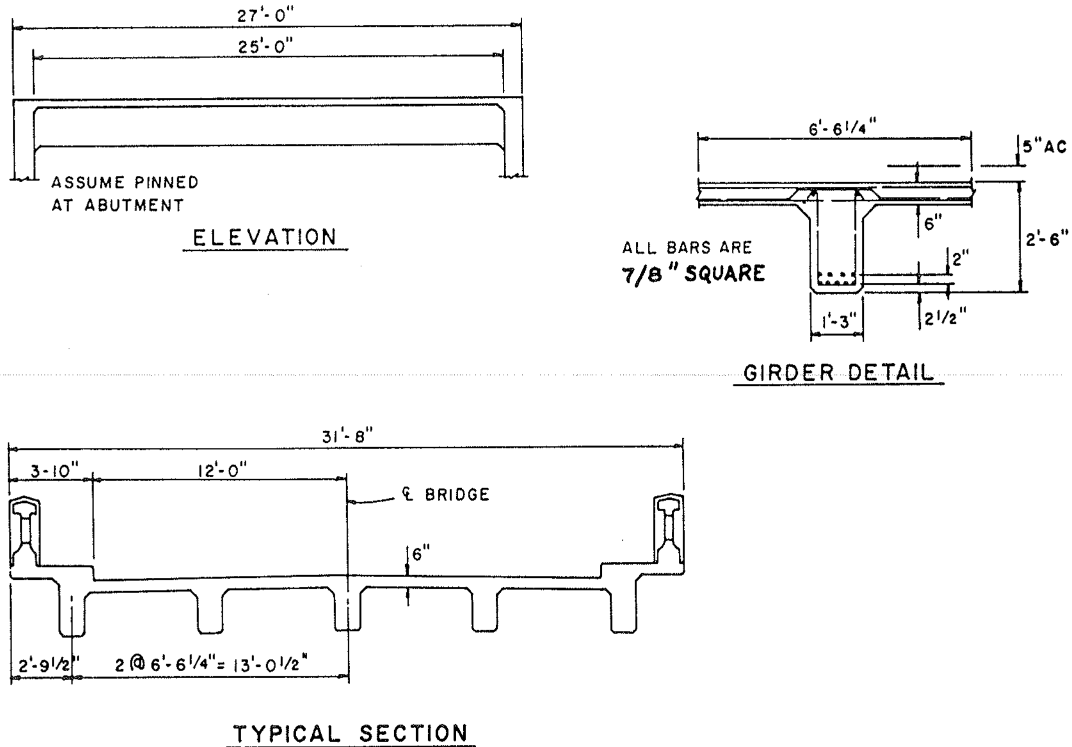

FIGURE 7 Single-span, T-beam bridge evaluated in example.

Different dead-load factors apply to each portion of the dead load. These factors will be selected from Figure 4, level 2, which is based on the tributary areas in which reactions are computed by including the continuity of the structure.

1. Total factored dead load:

$$\gamma_i^D \times w_{DL_i} = \gamma_i^D \times w_{DL_i} \tag{4}$$

Concrete section: 1.24 x 0.87 kip/ft = 1.08 kips/ft,
Rail: 1.31 x 0.22 kip/ft = 0.29 kip/ft, and
AC overlay: 1.50 x 0.40 kip/ft = 0.60 kip/ft.

Thus the total factored dead load = 1.97 kips/ft.

2. Dead-load moment (sum of dead-load effects):

$$\Sigma\gamma_i^D D_i = (\Sigma\gamma_i^D w_{DL_i} L^2)/8 = [1.97 \times (26)^2]/8 = 166 \text{ kip-ft} \tag{5}$$

Live-Load Factors

The live-load moments per wheel line of typical legal vehicles are taken from the AASHTO Manual for Maintenance Inspection of Bridges (1). The live-load moment for the HS20-44 truck was taken from the AASHTO Standard Specifications for Highway Bridges (7): type 3 = 93.5 kip-ft, type 3S2 = 90.2 kip-ft, type 3-3 = 77.0 kip-ft, and HS20-44 = 104.0 kip-ft.

Number of wheel lines = S/6 = 6.52/6 = 1.09 wheel lines.

The live-load distribution factors are based on formulas from AASHTO (level 2 live-load distributions from Table 6). In addition, the control of legal loads is vigorously enforced (category 1 live-load control from Table 7). Therefore, the live-load factor from Figure 5 is 1.50.

TABLE 7 Live-Load Control Categories

Load Limit Enforcement	Overload Sources	Category
Vigorously enforced: roadside weighing of trucks	Reasonable control of overloads at the source	1
Moderate enforcement of weight limits: no roadside weighing of trucks	Limited sources of overloads	2
Weight limits difficult to enforce	Many potential overload sources (mining, logging, and so forth)	3

Impact Factor

By using Equation 3, the calculated frequency is 15 Hz for a span length of 26 ft. The impact factor I = 0.30 is obtained from Figure 6.

Live Load Plus Impact Effect

Calculating the live-load moment effects for a typical interior girder with 1.09 wheel lines gives the following:

Type 3: $L_R(1.0 + I)$ = 93.5 x 1.09 x 1.30 = 132 kip-ft.
Type 3S2: $L_R(1.0 + I)$ = 90.2 x 1.09 x 1.30 = 128 kip-ft.
Type 3-3: $L_R(1.0 + I)$ = 77.0 x 1.09 x 1.30 = 109 kip-ft.
HS20-44: $L_R(1.0 + I)$ = 111.1 x 1.09 x 1.30 = 157 kip-ft.

Evaluation

The evaluation [rating factor (RF)] is calculated as follows:

$$RF = [(\phi R/\alpha) - \Sigma\gamma_i^D D_i]/[\gamma_R^L L_R (1.0 + I)] \tag{6}$$

The evaluation produces the following calculations:

Type 3: RF = [(0.94 x 494)/(1.0 - 166)]/(1.50 x 132) = 1.51.
Type 3S2: RF = [(0.94 x 494)/(1.0 - 166)]/(1.50 x 128) = 1.55.
Type 3-3: RF = [(0.94 x 494)/(1.0 - 166)]/(1.50 x 109) = 1.82.
Type HS20-44: RF = [(0.94 x 494)/(1.0 - 166)]/(1.50 x 157) = 1.27.

In order to compare the results, both the current AASHTO rating factors for HS20 loads that the California Department of Transportation calculated for this bridge by using the load-factor method and the rating factors that the Illinois Department of Transportation calculated by using the allowable stress method are given:

1. Inventory rating factor--California (load factor): RF = 0.97; and Illinois (allowable stress): RF = 0.86; and
2. Operating rating factor--California (load factor): RF = 1.61; and Illinois (allowable stress): RF = 1.44.

CONCLUSIONS

A proposed methodology for evaluating the structural strength of existing reinforced-concrete bridges that was developed in the first phase of an NCHRP project is presented. This methodology for rating bridges rationally combines probability theory with engineering judgment by using a limit-states format that contains both load and resistance factors. A somewhat general approach was taken in preparing the methodology so that all of the relevant variables, including some that have not yet been evaluated by scientific means, can be included in the rating process. The numerical values assigned to the load and resistance factors are based on limited statistical data and some preliminary calibration efforts.

The primary purpose of the proposed procedure is to place all of the variables involved in perspective so that they can be addressed, researched (if needed), and proportionally weighted in order that an overall evaluation procedure can be developed.

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